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Structural Evaluation and Testing of Mattawa River Bridge

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Abstract:

This report describes the structural evaluation and load test on the Mattawa River Bridge. The Mattawa River Bridge consists of two identical steel truss structures spanning the historic Mattawa River. The structural evaluation investigations showed that most of the main structural elements have adequate capacity to sustain the OHBDC loads without posting. A few selected members such as floor beams and portions of bottom chord and web members, however, were found to have slightly lower strength than required to carry OHBDC loads. The load tests performed on one of the bridge structures showed that the bridge floor system and the trusses have the reserve capacity to carry the OHBDC live loads without posting. It is recommended the bridge be repaired immediately to remedy the defects sighted in the recent condition survey report.

Key Words: testing, evaluation, Mattawa River Bridge, load testing, OHBDC load

Comments:

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1/ INTRODUCTION

When the canoe was the principal means of transport, explorers, voyageurs, missionaries and others used the Mattawa River as part of the river route link between the east and the west. Across this historic river now stands two identical steel truss bridges, carrying Highway 533, which were built in the year 1948. Typical of bridges around this district, they carry a substantial daily traffic volume of very heavy trucks transporting logged trees from forests to processing factories. A recent condition survey identified various structural problems in both the bridges, and analytical evaluations showed that some components of the bridges were substandard. In order to assure their continued usability, a proof load test of one of the bridge structures was performed in September 1983. This report presents the results of that investigation.

2/ BRIDGE DETAILS

The Mattawa River bridges consist of two identical steel truss structures, one spanning from the east bank of the river to the central embankment (shown in Figure 1) and the other spanning from the central embankment to the west bank of the river. The steel truss spans 32.9 m and has a 12.3 m wide concrete deck with two 4.57 m wide roadway lanes and a 1.5 m wide sidewalk, as shown in Figure 2. The deck consists of 178 mm concrete slab poured over a series of internal stringers (S15 x 42.9) and a pair of outer stringers (S12 x 35) all placed at 914 mm centres. Although the original drawings of the deck do not indicate their use, the structure now has approximately 80 mm asphalt topping and the associated waterproofing membrane. The sidewalk deck is 114 mm deep with no asphalt topping or waterproofing and is carried by two stringers (S10 x 20). The stringers are simply supported over floor beams (W33 x 152) located at each panel point of the truss. The truss generally consists of built-up members fabricated using steel angles, channels and/or plates and riveted connections. The truss has fixed bearings on one end of the structure and free bearings on the opposite end.

The bridge inspection study [1] conducted by the Proctor & Redfern Group, Consulting Engineers, identifies the following problems on both the bridge structures:

- very active corrosion of reinforcing steel within the concrete deck slabs;
- severe deterioration of concrete slab below the asphalt surface in some areas, poor waterproofing materials, poor concrete conditions at the sidewalk and curbs;
- leaking deteriorated deck expansion joints, rusted, seized expansion bearings; and
- rusted structural steel components.

The concrete is non air-entrained and the only sample tested showed a very low strength value of 16 MPa. The underside of the deck is reported to be in fairly good condition.

3/ BRIDGE EVALUATION

The bridge superstructure was evaluated according to the Ontario Highway Bridge Design Code [2]. The deck slab capacity (calculated using the empirical method given in Section 14) is approximately 800 kN, as opposed to 392 kN, the minimum required by the code for unposted bridge decks. In evaluating the deck load capacity, allowances were made for the lower concrete strength of 16 MPa given in the condition survey report and the fact that the deck is of non-composite construction.

The stringer and floor beam moments calculated for dead, superimposed dead and live loads are given in Table 1. The stringer dead and superimposed dead load moments were calculated by using simple beam analogy. The live load moments on stringers were calculated using grillage analysis. The floor beam moments were calculated by using the stringer reactions transferred onto the floor beams for various critical loading conditions. An impact factor of 0.3, the minimum required by the OHBDC, was used to calculate the ultimate live load effects. It is evident from the table that the stringers have adequate strength to sustain OHBDC loads while the floor beam is somewhat under strength for carrying the OHBDC live loads without posting.

For selected truss members the axial forces calculated for dead, superimposed dead and live loads are given in Table 2. A comparison of factored loads on the members to their respective factored member resistances is given in Table 3. An impact factor of 0.25 calculated as per OHBDC was used to evaluate the ultimate live loads on the truss members. The top chord members have adequate capacity to withstand the OHBDC loads for ultimate limit states (ULS). The bottom chord and web members for the most part have adequate capacity to carry the OHBDC loads, except for a few members which appear to have marginally lower strength than required. Since the truss members are generally composites of angles, channels and/or plates riveted together, the fact that they are marginally under-capacity is not critical. On the contrary, the members may sustain

forces well above the evaluated capacities. The longitudinal buckling load capacity of the truss was also evaluated using the computer program developed by Bakht and Csagoly [3]. The minimum single truck load capacity given by this program is approximately 1160 kN per truss. The total OHBDC truck load imposed on a truss at ultimate limit state using an impact factor of 0.25 is approximately 1225 kN. This means that the longitudinal buckling capacity of the truss is close to 95% of that required to sustain the ultimate OHBDC truck loads.

4/ INSTRUMENTATION AND TEST LOADING

The proof load testing of a bridge structure usually involves loading the structure to induce member forces to reach the levels of the ULS forces defined by the OHBDC. To study the various members' behaviour under applied loadings without overloading the bridge, the structure was instrumented substantially and monitored closely during the test. The bridge was instrumented using both weldable and demountable type uniaxial strain gauges and deflection transducers. The locations of strain gauges, deflection transducers used to monitor stringers, floor beams and north and south trusses are shown in Figure 3. Six stringers of the second panel were instrumented at their midspan locations. Similarly, three interior floor beams were instrumented at their midspan locations, as shown in Figure 3. For truss members, strain gauges were generally located at the centre point and the number of gauges attached varied depending on the critical nature of the member evaluated. In order to check for premature buckling of the cover plate caused by failure of top compression chord members [4], the most critical inclined chord member of the truss was instrumented with additional strain gauges both at the centre point and the end of the member. Vertical deflection transducers were attached to selected stringers, floor beams and to each panel point of the north truss. Transverse deflection transducers were used to monitor horizontal movements at both fixed and free bearing ends of the trusses.

The vehicular load during the test was applied by using two MTC test vehicles described by Bakht and Csagoly [5]. The two trucks were used both individually and together with gross loads of up to approximately 780 kN and then a single truck was used with a gross load of up to 890 kN. Two trucks both loaded to 890 kN were not used to test the bridge since it was thought that the probability of two heavily loaded trucks passing the bridge simultaneously causing a worse load effect would be negligible. It is shown in Holowka [6], based on the longitudinal moment effects on simple span lengths of 24.4 m and 31.4 m, that the test trucks loaded to 890 kN each are equivalent to the factored OHBDC truck loads to within 1%.

Since slightly lower truck loads (780 kN) were used for dual truck loading cases studied, the induced member forces may not reach the levels of the ULS from the OHBDC on all members. The applied loading, however, is considered to be realistic for the bridge under study.

The strain gauges and deflection transducers were installed a few weeks prior to the testing of the bridge. The weights of vehicles were gradually increased and for each load level the trucks were placed either individually or together at predetermined locations, which are shown in Figure 4. The data from the strain gauges and deflection transducers were recorded through a computer based multi-channel data acquisition system described by Bakht and Csagoly [5]. During the test, no unduly high strains were recorded in any of the gauges monitored and the strains recorded were reversible.

5/ TEST RESULTS

The bridge structure carried the applied truck loads without showing visible signs of distress. This in itself suggests that the bridge can carry the OHBDC live loads without posting. The strain and deflection measurements made during the tests were analysed, however, to identify possible weaknesses in the structural elements which remained undetected. This investigation is divided into the following groups:

- stringers
- floor beams
- truss members
- temperature study

5.1/ Stringers

Typical stringer beam strain diagrams plotted using the strain measurements from the test are illustrated in Figure 5. The strain diagrams exhibit some degree of composite action since the bottom flange strains are observed to be very much higher than the top flange strains. The deck construction is non-composite and hence it is deduced that the beam must be behaving as a partially composite deck.

The theoretical stringer moments, calculated using grillage analysis of the deck under applied loading, and the moments, calculated using the strain measurements for the non-composite or the composite section properties, are shown plotted against the stringer locations in Figures 6 and 7. Loading cases with trucks loaded to approximately 780 kN and 890 kN are illustrated in Figure 6 and 7 respectively. For the grillage analysis of the deck, the non-composite deck properties are used. Based on these figures, the shape of the load distribution graphs obtained using the measured strains are quite similar to those calculated using grid analogy. However, the theoretical moment plots lie between the upper and lower bound moment values calculated using non-composite and fully composite section properties. This also confirms that the stringer sections are acting partially composite with the concrete deck. The

bridge evaluation shows that the stringers have adequate capacity to sustain the OHBDC loads and the test results confirm that the stringers act as a partially composite beam. From these observations, it is concluded that the stringers have strength well in excess of that required by OHBDC.

5.2/ Floor Beams

The strain measurements made on the floor beams are shown plotted in Figure 8 for loading cases with truck loads of 780 kN and 890 kN. The bottom flange strains are generally greater than the top flange strains measured, showing that the floor beam exhibits some degree of composite action with the concrete deck. The level of interaction between the concrete deck and the floor beam, however, is much less than that observed for the stringer.

The moments on the floor beams, calculated using the stringer reactions obtained from the grid analysis, are given, related to the respective moments derived from the strain measurements and floor beam non-composite section properties, in Table 4. The load transferred to the floor beams is usually obtained without allowing for the possible load sharing effects developed by the longitudinal stiffness of the stringers, the concrete deck and/or the floor beam flexibility. Bakht [7] has developed a technique which gives correction factors for moments on the floor beams obtained by simplified methods to account for the load sharing effect between the adjacent floor beams. Table 4 gives the values of moments which have been corrected for the load sharing effect of the floor beams. For all the floor beams, the values thus obtained compare very well with the measured moments for each of the loading cases. Under the applied test truck loads, the floor beams behaved linear-elastically and the measured moments were about 75% of the uncorrected calculated moments (moments usually calculated for the design/evaluation of floor beams). This confirms that the floor beams have substantial reserve capacity for live loads greater than those applied by the test trucks and therefore can safely carry the OHBDC live loads without posting.

The vertical deflection results are somewhat suspicious because of transducer tensioning wire movements caused by floating weeds and the river

current. The deflections measured under floor beams marked 2 and 3 are given in Table 5. Generally, the measured deflections are found to be less than the theoretical midspan deflections.

5.3/ Truss Members

The trusses supporting the roadway deck and the sidewalk area act as a single determinate structure and hence, with the exception of the vertical members, require all the members to actively participate in dissipating the applied loads to the end supports. The verticals provide lateral restraint to the top closed members. Failure of one member can prove to be disastrous to the bridge structure. The truss members were instrumented comprehensively, therefore, to safely load test the bridge and to study the various member behaviours under applied loadings.

Typical stress variations observed in the components of the built-up truss members under various loading cases are illustrated in Figure 9. At the middle section of the inclined end chord member, the stresses in the channels are found to be substantially higher than the stresses on the plate section. However, the gauges attached at the end section of the same inclined chord show that the plate stresses are greater than the flange section stresses. The middle sections of all other top chords usually show a higher stress in the plate than in the channel sections. The stress distributions in the components of the bottom chord members are generally found to be approximately equal. Although some web members exhibited unequal load distribution between the inner and outer elements, it was found that for the most part the loads in the web components were distributed approximately equally.

Due to the variations of stress in the elements of the built-up sections, especially for the inclined end chord and/or top chords, the actual member forces induced by the test truck loads were obtained by summing the forces calculated for each element. The forces on each element were calculated using the average stress on the section and the corresponding cross-section. For selected top chord, bottom chord and web members, the member forces calculated for a pin connected truss and using the applied truck loadings are given in comparison to the member forces obtained from

measured stress values in Table 6. Two bridge loading cases were used to prepare this table:

1/ two trucks each weighing 780 kN; and

2/ a single truck with a load of 890 kN.

The measured top chord member forces were usually lower than the calculated member forces. Measured values ranged between 80 and 100% of the calculated values. The only adequate explanation for this is that the bridge as a system generally exhibits greater load sharing and load distribution between its members than considered usual in theoretical analysis.

The bottom chord members gave measured values much lower than the corresponding calculated member forces. The measured member forces were generally 50% below the calculated values. The bottom chord nearest to the bearings showed a compressive force, although the calculated force is obviously tensile. It is thought that the reduction in forces on the bottom chords and/or the compressive force exhibited on the bottom chord nearest to the bearings are due to:

1/ the stringers and concrete deck sharing the tensile forces transferred to the bottom chord; and

2/ the free bearing movements becoming restricted by rusting and accummulation of dirt and debris.

An analysis of the truss was performed assuming that the bearings on both its ends were fixed. As expected, the top chord and web member forces were found to be unaffected, but the bottom chord forces were altered, see Table 7. The bottom chord nearest to the bearings is now required to carry considerable compressive force. The measured compressive force, however, was very much lower than the theoretical value. Since most of the bottom chord members were incapable of carrying such a high compressive forces they may have flexed and, as a result, shown a lower measured value. If the bearings were fully fixed as assumed, it was also possible that part of the compressive load was shared by the deck and/or stringers. Surprisingly, for the bottom chord members near the centre span, forces calculated by assuming fixed bearings are very close to the

measured values. Nevertheless, rather than repeating the test with bearings confirmed to be operational (free), it is concluded here that the observed reduction in bottom chord forces are due to the combined effects of:

1/ loading sharing between bottom chord, stringers and/or deck; and

2/ semi-rigid bearings.

It is suggested that in a future test of a similar bridge, the effects of semi-rigid and free bearings be studied closely.

The end web members gave measured member forces close to the calculated member forces but the former values were generally lower than the latter for the webs located toward the middle of the bridge.

Although some of the vertical deflection measurements were affected by floating weeds and the river current, the truss deflections from a bridge loading of two trucks of 780 kN measured at the panel points were not affected. These are compared to the deflections calculated assuming free bearings in Table 8. The measured vertical deflections are usually lower than the calculated values. Although deflections calculated assuming fixed bearings were generally greater than measured deflections, the difference between these values was considerably reduced, see Table 8b. There was practically no movement recorded at the fixed end of the bridge. Although the free end bearings showed movements, the measured values were much lower than the calculated values. The calculated deflections assuming free bearings are usually greater than the theoretical deflections because:

- the bearings became semi-rigid after years of service and inadequate maintenance; and
- the bridge truss acting as a structural system has considerably higher rigidity than usually modelled theoretically.

The truss and its members, however, behaved linear-elastically under the applied truck loads, which could have caused force effects quite close to OHBDC level 3 loads. The stresses recorded in testing confirm that the truss elements have greater capacity than required to sustain the test truck loads and hence the OHBDC loads without posting.

5.4/ Temperature Study

The truss movement/member strains and the temperature variations were monitored for a period of 25 h to:

- investigate the functionality of the truss' free bearings, and
- check any undue stress build-up in the truss members.

During this period, the air temperature varied approximately 6°C. Figure 10 illustrates the temperature variations observed and the lateral movements measured at the expansion joint. As expected, the air temperature leads the temperature measured on the structure. The truss movements graph and the steel temperature gradient graph appear to be of an identical shape after a time ordinate of 10 h. However, the truss movement observed before 10 h does not correlate with the observed temperature change. Strains measured on truss members were very low and confirm that there is no undue stress build-up in the members due to temperature change. The calculated end movement for the observed steel temperature variation of 3.5°C is 1.34 mm while the measured movement is 0.5 mm. The observed difference between the calculated and measured values could be due to the following:

1/ the bearings may not function satisfactorily,

2/ the bridge, as a system may undergo much less movement than calculated theoretically.

It is recommended that the bearings be checked immediately and repaired if necessary.

6/ CONCLUSIONS AND RECOMMENDATIONS

The bridge evaluation study carried out as per the OHBDC reveals the following:

- The bridge deck slab has considerable reserve capacity to take OHBDC live loads.

- The strength values evaluated for the stringers are greater than those induced by the factored OHBDC loads, while the floor beams have marginally lower strength than required.

- The truss top chord members have compressive resistance values well over those required for the OHBDC factored loads.

- The truss bottom chord and web members have sufficient member resistances to sustain OHBDC loads, except for a few isolated members that have slightly lower resistance values than required.

- The buckling load capacity of each truss is 95% of the factored total load of a single OHBDC truck.

The load test performed on the structure shows that the stringer beams and the concrete deck behave as a partially composite section. The theoretical moment values obtained using grillage analysis are shaped similarly and lie between the upper and lower bounds of the moment value plots obtained using the measured strains, non-composite and composite section properties respectively. The stringer-deck system has sufficient capacity to take OHBDC live loads without posting because:

- the evaluation study showed that the stringers, acting non-compositely, have strength values greater than required to sustain OHBDC factored loads;
- the bridge carried very heavy vehicles and the stringers were well within the elastic stress range; and
- the stringers act as a partially composite beam.

The floor beams also showed partially composite action with the concrete deck, however the extent of composite action was much less than that observed for the stringer-concrete deck elements. The calculated moments, corrected for the effects of load sharing between floor beams, compare very well with the moment values evaluated using the measured strains on the bottom flange and the floor beams' elastic modulus values. Under the applied loadings, the floor beams were within the elastic stress range. The measured moments were approximately 76% of the uncorrected theoretical moments usually used for design and evaluation of floor beams. This confirms that the floor beams have substantial reserve capacity to take Ontario legal loads without posting.

The stresses imposed on the built-up components of the truss were found to be non-uniform. This was highly pronounced in the end-inclined and top chord members. For the bottom chord and web members, however, the stress distribution was generally found to be uniform. The measured top chord values were lower than the theoretical ones, the former ranging between 80 and 100% of the latter. The bottom chord members, on the other hand, gave measured values much lower than the theoretical values. The bottom chord nearest to the bearings, in fact, gave compressive instead of tensile force values. For the other bottom chords, the measured values were generally 50% below the theoretical values. An analysis conducted with fixed bearings showed compressive forces only on the bottom chord nearest to the bearings and gave tensile force values very close to the measured tensile force values for the rest of the bottom chord members. Without repeating the test with bearings proven to be functioning normally, it is concluded that the bottom chord force reduction is due to:

1/ defective bearings, and

2/ the effect of load sharing between the bottom chord, stringers and/or the concrete deck.

It is suggested that in future tests on a similar bridge the effects of fixed bearings on bottom chord forces be studied closely.

The end web members showed measured member forces quite close to the calculated member forces, except toward the middle of the truss where the former values are lower than the latter. The load tests indicated that

all the truss members were well within the elastic stress range and the measured member forces were generally lower than the calculated member forces (the degree of reduction depended on the member type). The above findings confirm that the truss has adequate capacity to carry unrestricted Ontario legal loads. A few deflection transducers may have given erroneous results due to the tensioning wire movements caused by weeds and/or the river current. The measured deflections for the floor beams are lower than the calculated deflection values in most cases. The truss showed calculated values well above the measured deflection values. Deflections, calculated assuming fixed bearings, reduced the difference between the calculated and measured values.

Due to the limited range of the temperature and the free end bearing movements recorded, the question of the functionality of the roller bearings remains inconclusive.

Although the bridges are found to be structurally adequate, it is recommended that they be repaired immediately to remedy the defects revealed in the recent condition survey report. The free end bearings of the trusses require close inspection and any necessary corrections should be made.

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Table 1a/ Unfactored Moments on Stringers and Floor Beams

Member	Maximum Moments, kNm		
	Dead Load	Super-imposed Dead Load	Live Load
Stringer	12.5	4.5	59.3
Floor Beam	325.5	106.4	831.9

Table 1b/ Factored Moments and Resistances on Stringers and Floor Beams

Member	Factored Moments, kNm	Factored Resistances, kNm	
		Laterally Supported	Laterally Unsupported
Stringer	129.7	202.0	175.0
Floor Beam	2080.5	1652.0	——

Table 2/ Factored Axial Forces on Selected Truss Members

Member	Factored Axial Forces, kN			
	Dead Load	Super-imposed Dead Load	Live Load	Total
Top Chord				
1 - 2	-852.3	-271.6	-1301.8	-2425.7
2 - 5	-1023.0	-326.1	-1562.2	-2911.3
5 - 8	-1383.8	-441.0	-2204.4	-4029.2
8 - 11	-1498.1	-477.4	-2204.4	-4179.2
Bottom Chord				
1 - 3	572.6	182.4	871.6	1626.6
3 - 6	1207.6	384.9	1891.2	3433.7
6 - 9	1373.3	437.7	2046.5	3857.5
9 - 12	1498.1	477.4	2020.0	3999.5
Web				
2 - 3	654.3	212.1	996.2	1862.6
3 - 5	-334.9	-106.8	-675.4	-1117.1
5 - 6	277.6	88.4	675.4	1041.4
6 - 8	-7.5	-3.0	453.9	443.4
8 - 9	244.6	77.9	-453.9	-131.4

- Compressive

+ Tensile

Table 3/ Factored Member Forces Related to Factored Member Resistances of Selected Truss Members

Member	Member Forces, kN	Member Resistances, kN
Top Chord		
1 - 2	2425.7	2716.0
2 - 5	2911.3	4168.0
5 - 8	4029.2	4168.0
8 - 11	4179.9	4542.0
Bottom Chord		
1 - 3	1626.6	1620.0
3 - 6	3483.7	3245.0
6 - 9	3857.5	3988.0
9 - 12	3999.5	3988.0
Web		
2 - 3	1862.6	1751.0
3 - 5	1117.1	904.0
5 - 6	1041.4	1493.0
6 - 8	443.4	858.0
8 - 9	131.4	359.0

Table 4/ Calculated Floor Beam Moments Related to Measured Floor Beam Moments for Various Loading Cases

Loading Case	Calculated Maximum Floor Beam Moments, kNm	Floor Beam Midspan Moments, kNm		
		Uncorrected	Corrected for Load Sharing	Measured
Floor Beam 1				
Level4/Line1/Pos.1	236.8	197.1	147.8	158.5
Level4/Line2/Pos.1	514.1	514.1	385.5	391.9
Level4/Line3/Pos.1	752.5	713.1	534.8	554.5
Level4/Line4/Pos.1	783.4	782.9	584.2	604.8
Level5/Line1/Pos.1	281.0	233.9	175.4	186.4
Level5/Line2/Pos.1	603.2	603.2	452.4	456.9
Floor Beam 2				
Level4/Line1/Pos.3	236.8	197.1	147.8	156.4
Level4/Line2/Pos.3	514.1	514.1	385.5	383.0
Level4/Line3/Pos.3	752.5	713.1	534.8	539.0
Level4/Line4/Pos.3	783.4	782.9	584.2	604.0
Level5/Line1/Pos.3	281.0	233.9	175.4	170.0
Level5/Line2/Pos.3	603.2	603.2	452.4	434.0
Floor Beam 3				
Level4/Line1/Pos.4	236.8	197.1	147.8	172.5
Level4/Line2/Pos.4	514.1	514.1	385.5	420.2
Level4/Line3/Pos.4	752.5	713.1	534.8	585.8
Level4/Line4/Pos.4	783.4	782.9	584.2	661.5
Level5/Line1/Pos.4	281.0	233.9	175.4	189.3
Level5/Line2/Pos.4	603.2	603.2	452.4	489.1

Table 5/ Measured and Calculated Floor Beam Deflections

Load Case	Theoretical Midspan Deflections, mm	Measured Midspan Deflections, mm
Floor Beam 2		
Level4/Line1/Pos.2	3.1	3.9
Level4/Line2/Pos.2	6.9	5.1
Level4/Line3/Pos.2	10.1	6.7
Level4/Line4/Pos.2	11.3	6.9
Level5/Line1/Pos.2	3.7	1.8
Level5/Line2/Pos.2	8.1	4.9
Floor Beam 3		
Level4/Line1/Pos.4	3.1	3.3
Level4/line2/Pos.4	6.9	5.9
Level4/Line3/Pos.4	10.1	7.0
Level4/Line4/Pos.4	11.3	6.3
Level5/Line1/Pos.4	3.7	4.2
Level5/Line2/Pos.4	8.1	5.2

Table 6a/ Member Forces on the North Truss due to Trucks Loaded to 780 kN.

MEMBER	LINE 3								LINE 4							
	Pos. 1		2		3		4		Pos. 1		2		3		4	
	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.
Top Chord																
1 - 2	998.7	740.4	900.0	737.7	790.7	668.4	592.9	523.9	746.9	563.4	682.6	561.9	597.5	511.3	447.9	403.2
2 - 5	1187.1	998.7	1080.7	1004.7	949.3	916.8	711.9	721.9	896.9	759.9	819.6	770.7	717.4	706.7	537.8	559.6
5 - 8	1468.0	1099.2	1573.1	1227.0	1565.9	1290.5	1149.6	1097.0	1109.4	851.2	1194.9	942.8	1183.2	981.6	868.4	846.5
8 - 11	1358.5	1079.3	1542.9	1284.0	1665.0	1419.0	1586.5	1447.4	1028.3	819.6	1169.8	983.7	1258.2	1103.1	1198.9	1102.7
Bottom Chord																
1 - 3	664.4	-3.1	604.8	-62.5	531.3	-148.4	398.4	-242.2	501.9	-20.5	458.7	-37.1	401.5	-106.9	301.0	-171.3
3 - 6	1325.7	452.2	1335.8	457.4	1273.8	417.3	941.7	233.0	1001.7	365.2	1014.1	365.8	962.5	330.1	711.4	202.9
6 - 9	1341.1	519.3	1480.7	586.3	1538.6	619.6	1311.5	535.3	1014.3	414.2	1123.6	471.0	1162.5	492.2	990.9	439.9
9 - 12	1202.1	427.8	1414.5	545.0	1599.4	655.0	1625.4	715.0	909.1	345.1	1071.1	444.6	1209.3	525.3	1228.1	558.4
Web																
2 - 3	759.2	636.1	691.1	665.5	607.2	611.5	455.3	463.2	573.6	503.6	524.2	502.4	458.8	469.0	344.0	364.5
3 - 5	259.2	174.4	457.3	276.7	573.2	428.5	407.2	422.8	196.2	147.7	348.6	225.1	433.5	328.9	307.5	325.7
5 - 6	216.7	115.5	377.5	189.5	471.7	307.2	335.0	312.5	164.0	95.6	287.8	150.2	356.4	228.7	253.0	233.7
6 - 8	217.3	176.7	147.4	100.5	19.9	41.2	341.9	273.5	162.6	134.0	114.3	69.1	15.2	3.1	258.7	203.8
8 - 9	34.1	9.7	122.0	72.4	248.4	131.2	538.8	331.0	27.4	6.5	90.4	63.1	187.5	120.2	407.5	251.0

Table 4b/ Member Forces on the North Truss due to Truck Loaded to 890 kN

MEMBER	LINE 1								LINE 2							
	Pos. 1		2		3		4		Pos. 1		2		3		4	
	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.
Top Chord																
1 - 2	707.1	538.8	644.9	528.3	565.6	471.7	426.2	362.7	432.3	317.9	392.0	317.5	346.2	285.4	260.2	229.4
2 - 5	849.0	715.7	744.4	707.8	679.1	637.2	511.7	503.1	519.0	437.6	470.7	443.7	415.7	406.4	312.4	330.8
5 - 8	1051.9	783.6	1121.9	883.5	1122.2	928.3	825.9	774.4	626.4	490.8	684.2	540.5	684.5	562.2	504.8	486.1
8 - 11	966.1	761.7	1095.9	925.1	1193.2	1008.8	1142.4	1036.3	582.0	475.4	666.9	568.1	715.5	628.1	695.2	624.8
Bottom Chord																
1 - 3	475.1	-3.70	433.4	-55.5	380.1	-119.2	286.4	-177.6	290.5	-11.3	263.4	-23.3	232.6	-35.4	174.8	-98.5
3 - 6	949.2	322.9	954.3	328.2	912.3	295.2	676.7	163.1	571.2	215.2	581.3	216.9	557.1	194.4	413.4	123.9
6 - 9	957.3	371.9	1053.8	425.6	1102.4	449.8	943.5	390.0	573.4	242.8	641.9	277.1	666.4	291.3	575.2	253.0
9 - 12	855.4	299.7	1004.8	400.8	1142.3	478.9	1170.9	523.2	514.8	201.2	611.3	260.1	687.8	313.7	706.2	327.1
Web																
2 - 3	543.0	485.7	495.2	485.7	434.3	437.9	327.3	333.0	331.9	283.4	301.0	285.5	265.9	269.3	199.8	210.0
3 - 5	187.3	113.9	322.8	190.1	412.4	316.7	292.3	302.4	98.9	62.7	198.2	119.3	250.1	180.9	179.0	175.2
5 - 6	156.5	71.6	266.5	120.5	339.0	229.2	240.4	222.4	83.0	55.7	163.7	66.3	205.7	119.4	147.2	117.3
6 - 8	163.0	131.8	109.5	70.8	14.8	36.0	248.2	202.3	90.4	73.6	68.2	42.1	20.9	5.9	148.8	118.5
8 - 9	17.2	9.1	82.6	52.2	177.4	88.3	389.6	247.2	16.9	4.9	49.0	33.8	96.4	75.7	235.3	146.3

Table 7/ Bottom Chord Forces Calculated Assuming Fixed Bearings Compared to Measured Bottom Chord Forces

	LINE 3							
BOTTOM CHORD	1		2		3		4	
	Cal. kN	Meas. kN	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.
1 - 3	-119.5	-3.1	-257.9	-62.5	-374.9	148.4	-522.6	-242.2
3 - 6	521.7	452.2	477.7	457.4	381.0	417.3	72.4	233.0
6 - 9	557.4	519.3	625.7	586.4	632.5	619.6	441.5	535.3
9 - 12	418.2	427.8	563.9	545.0	687.0	655.0	735.6	715.0

Table 8a/ Measured and Calculated Movements of Truss Under Two Trucks Loaded to 780 kN

DISPLACEMENT GAUGE NO.	LINE 3								LINE 4							
	Pos. 1		2		3		4		Pos. 1		2		3		4	
	Cal. (mm)	Meas. (mm)	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.
Vertical Displacement Transducer																
84	17.4	8.2	17.2	9.4	16.3	8.8	13.1	6.9	13.2	6.8	13.0	6.9	12.4	6.4	9.9	5.9
85	28.3	13.9	30.5	15.0	31.0	16.3	24.7	13.3	21.3	11.1	23.1	12.8	23.4	13.3	18.6	11.3
86	30.3	14.2	34.4	15.6	37.4	17.6	36.2	18.9	22.9	11.3	26.0	13.7	28.2	15.5	27.3	14.9
87	25.3	9.6	30.3	9.7	35.3	14.4	37.4	12.9	19.1	5.4	22.9	5.1	26.7	6.8	28.3	8.2
88	17.5	8.1	20.9	9.8	24.8	12.6	30.2	15.0	13.2	5.9	15.8	8.2	18.7	10.9	22.8	11.8
89	9.2	3.3	11.0	4.4	13.1	5.6	16.2	7.6	6.9	2.4	8.3	3.2	9.9	4.3	12.2	5.8
Horizontal Displacement Transducers																
93	0	0.04	0	0.07	0	0.07	0	0	0	0.04	0	0.03	0	0.03	0	0.03
94	11.1	1.09	12.1	1.29	12.9	1.40	12.9	1.47	8.4	1.19	9.2	1.35	9.8	1.45	9.8	1.47

Table 8b/ Measured and Calculated Movements of Truss with Fixed Bearings at Both Ends and Under Two Trucks Loaded to 780 kN

	LINE 3							
DISPLACEMENT GAUGE NO.	Pos. 1		2		3		4	
	Cal. (mm)	Meas. (mm)	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.
VERTICAL DISPLACEMENT TRANSDUCER								
84	11.2	8.2	10.4	9.4	9.6	8.8	6.6	6.9
85	19.2	13.9	20.7	15.0	20.8	16.3	15.6	13.3
86	19.9	14.2	23.3	15.6	25.6	17.6	25.3	18.9
87	15.0	9.6	19.3	9.7	23.4	14.4	26.2	12.9
88	8.4	8.1	11.3	9.8	14.3	12.6	20.1	15.0
89	3.0	3.3	4.4	4.4	5.9	5.6	9.2	7.6
HORIZONTAL DISPLACEMENT TRANSDUCER								
93	0	0.04	0	0.07	0	0.07	0	0
94	0	1.09	0	1.29	0	1.40	0	1.47

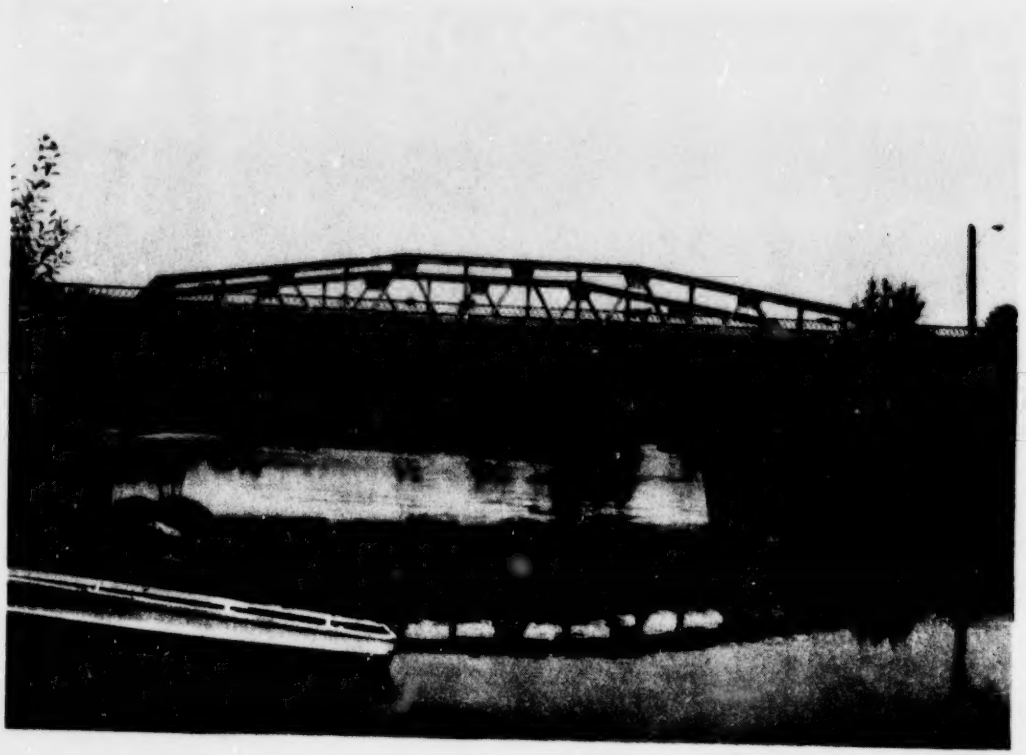
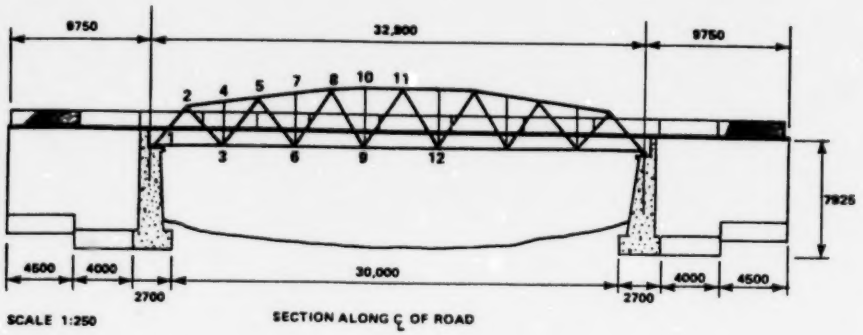
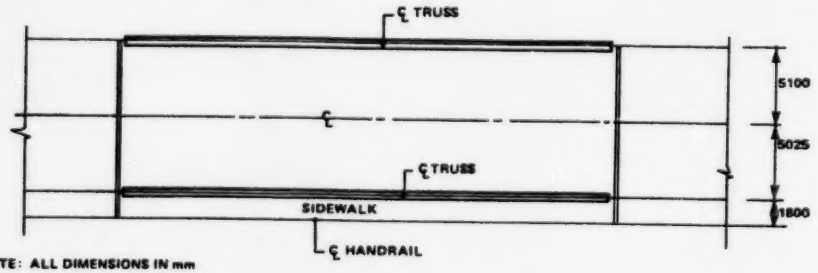


Figure 1/ A View of Mattawa River Bridge (The East Structure)

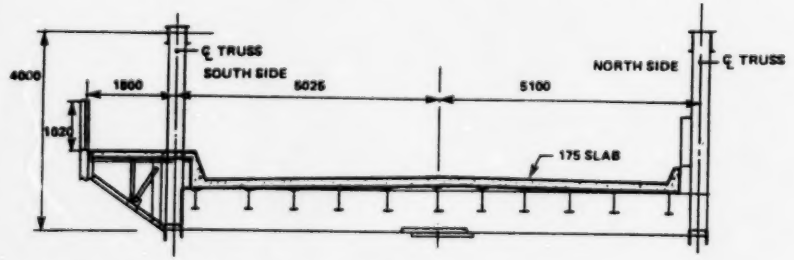


SCALE 1:250 SECTION ALONG ℄ OF ROAD



NOTE: ALL DIMENSIONS IN mm

SCALE 1:250 PLAN VIEW

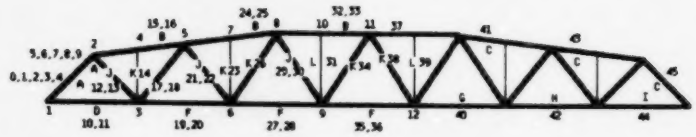


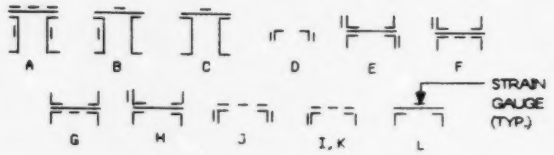
SCALE 1:75 SECTION OF FLOOR SYSTEM

Figure 2/ Bridge Details

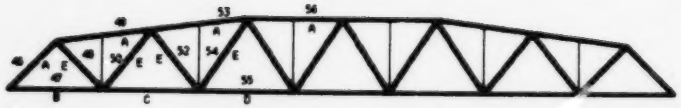
Member Details

MEMBER	MEMBER SIZES (Imperial Units)
Top Chord	
1 - 2	2[S,12"@25; 1PL18" x 3/8"
2 - 5	2[S,12"@35; 1PL18" x 3/8"
5 - 8	2[S,12"@35; 1PL18" x 5/8"
8 - 11	2[S,12"@40; 1PL18" x 5/8"
Bottom Chord	
1 - 3	2[S,6" x 3 1/2 x 9/16"
3 - 6	4[S,6" x 3 1/2 x 11/16"
6 - 12	4[S,6" x 3 1/2" x 3/4"; 1PL, 10 1/2 x 3/8
Web	
	2[S,6" x 3 1/2" x 9/16"; LACED
3 - 4; 6 - 7; 9 - 10	2[S,5" x 3" x 1/2"; LACED
3 - 5	2[S,6" x 3 1/2" x 9/16"; LACED
5 - 6	2[S,5" x 3 1/2" x 3/8"
6 - 8	2[S,5" x 3" x 3/8"; LACED
8 - 9	2[S,5" x 3" x 3/8"
9 - 11	2[S,5" x 3" x 3/8"; LACED

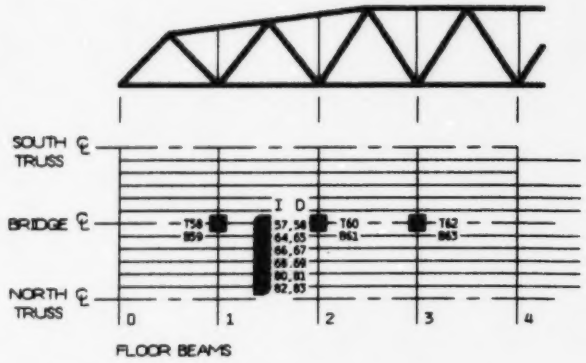


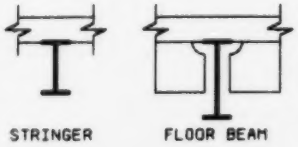


LOCATION OF STRAIN GAUGES ON NORTH TRUSS

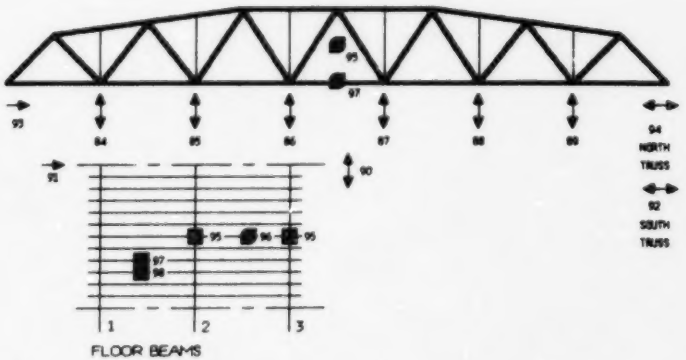


LOCATION OF STRAIN GAUGES ON SOUTH TRUSS



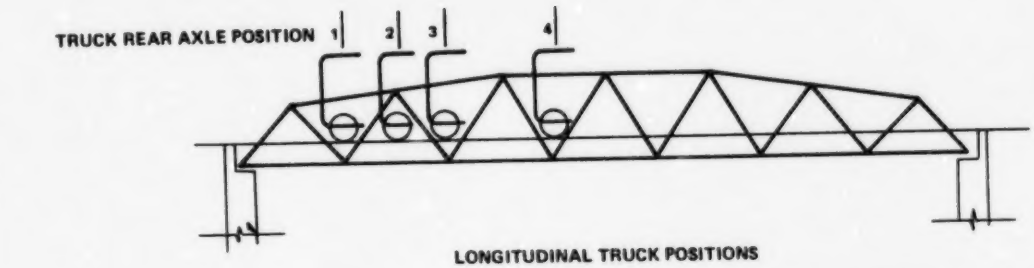


LOCATION OF STRAIN GAUGES STRINGERS/FLOOR BEAMS

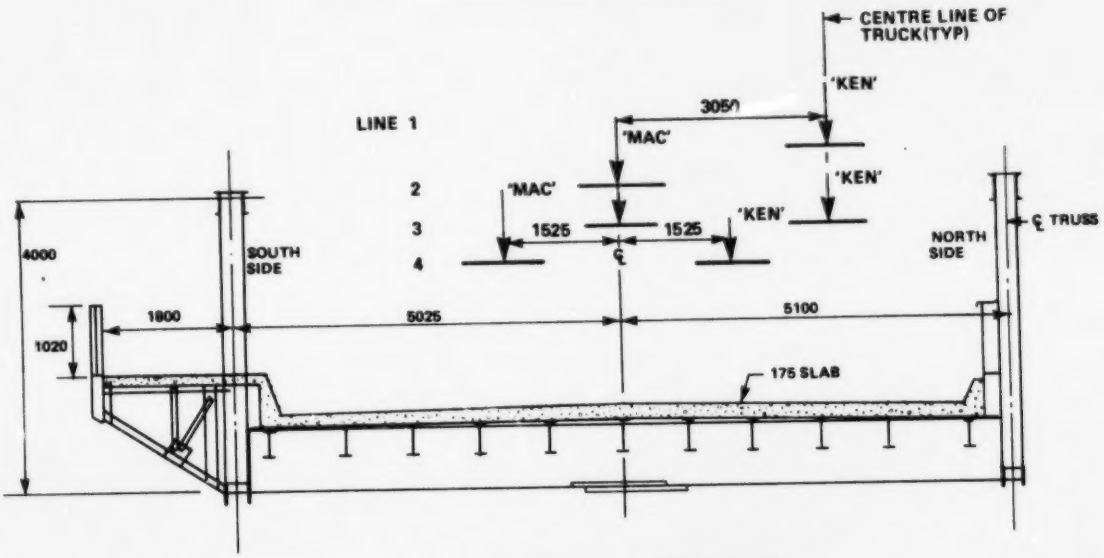


DEFLECTION TRANSDUCER/THERMOCOUPLES ATTACHED ON THE TRUSS, STRINGER AND FLOOR BEAMS

Figure 3/ Instrumentation of the Bridge



LONGITUDINAL TRUCK POSITIONS



TRANSVERSE TRUCK POSITIONS

AXLE WEIGHTS OF THE TEST VEHICLES

	Load Lift No.	Weight of Axle No. (kN) 1	2	3	4	5	Total Vehicle Weight (kN)
'KEN'	24	47.7	92.7	92.7	105.2	105.2	443.5
	36	49.4	115.5	115.5	136.9	136.9	554.2
	48	51.7	138.7	138.7	168.9	168.9	666.9
	60	53.5	161.9	161.9	201.1	201.1	779.5
	72	56.3	180.5	180.5	237.7	237.7	891.7
'MAC'	24	43.6	90.1	90.1	104.7	104.7	433.2
	36	44.8	113.2	113.2	137.2	137.2	545.4
	48	45.9	136.0	136.0	170.3	170.3	658.5
	60	46.8	159.2	159.2	203.3	203.3	779.5

Figure 4/ Truck Positions and Loading Used for the Tests

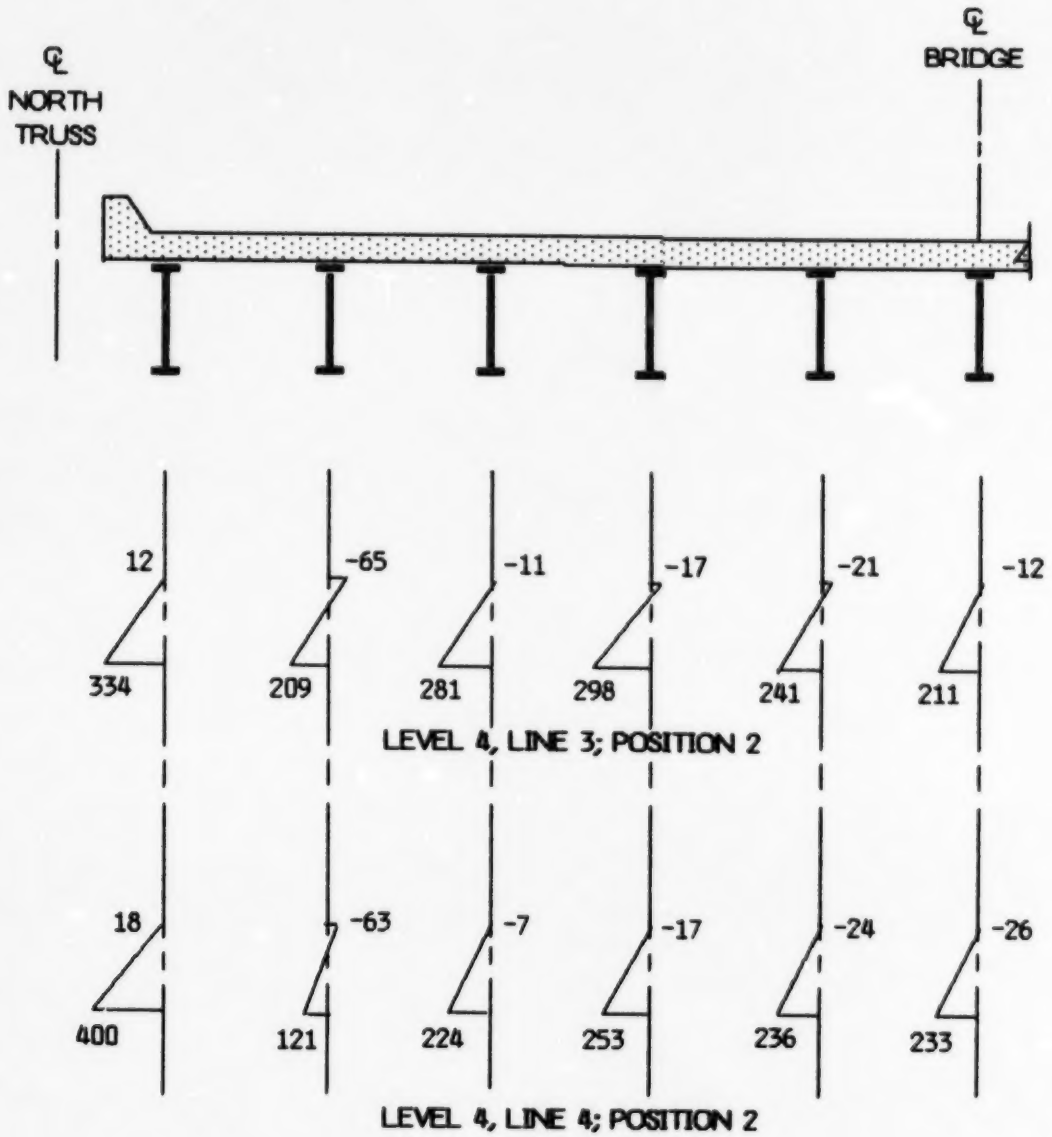


Figure 5/ Typical Strain Diagrams for Stringers

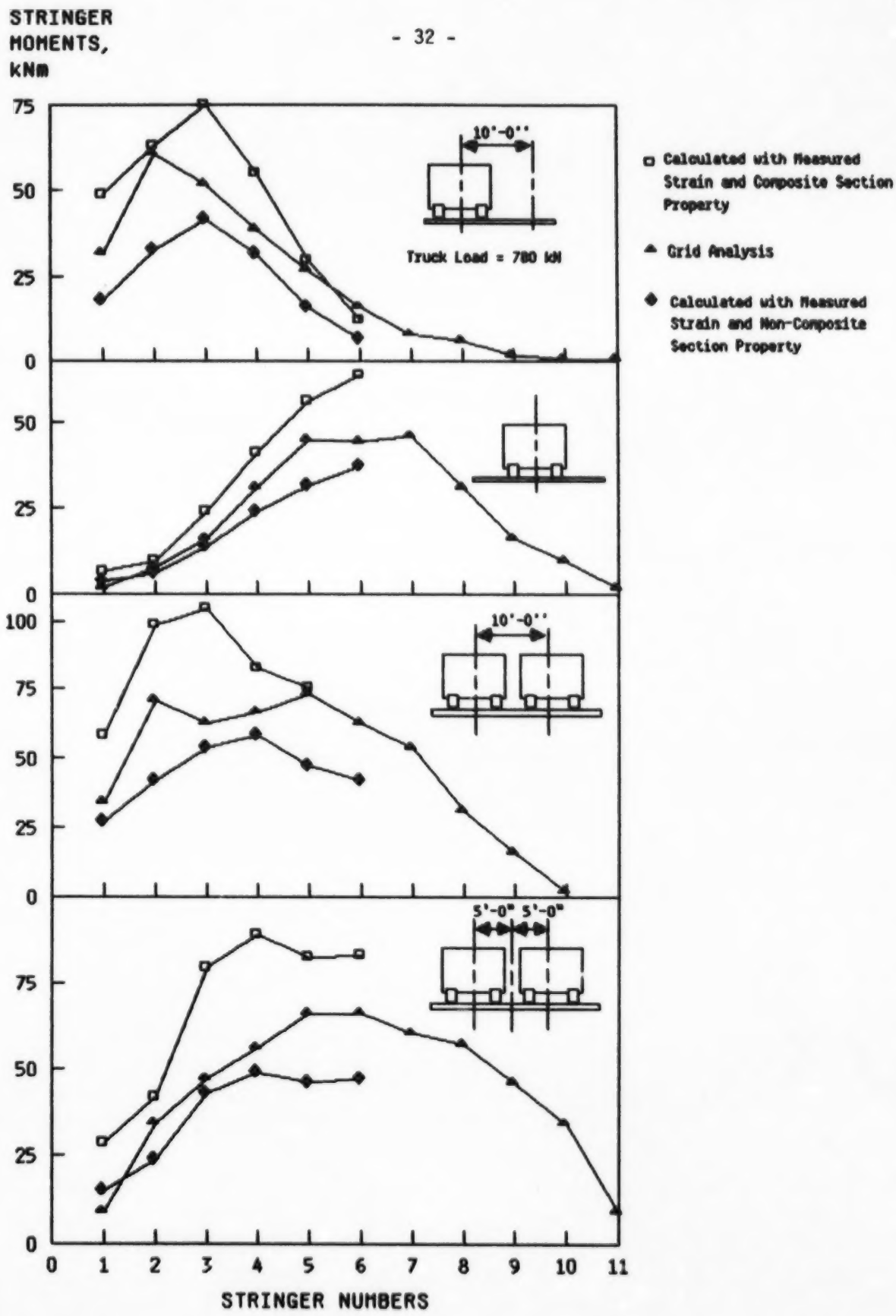


Figure 6/ Theoretical Stringer Moments Related to Measured Stringer Moments

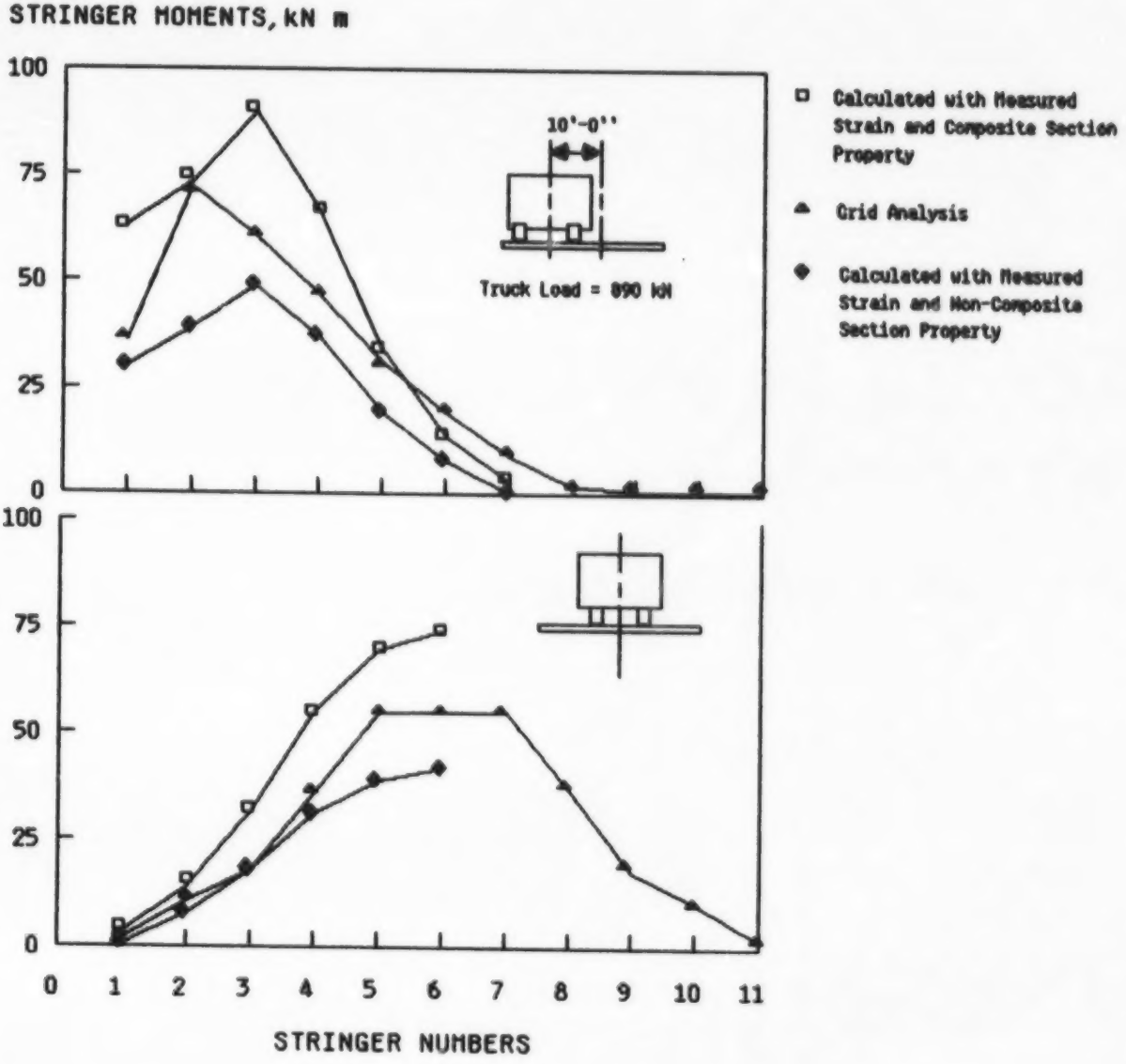


Figure 7/ Theoretical Stringer Moments Related to Measured Stringer Moments

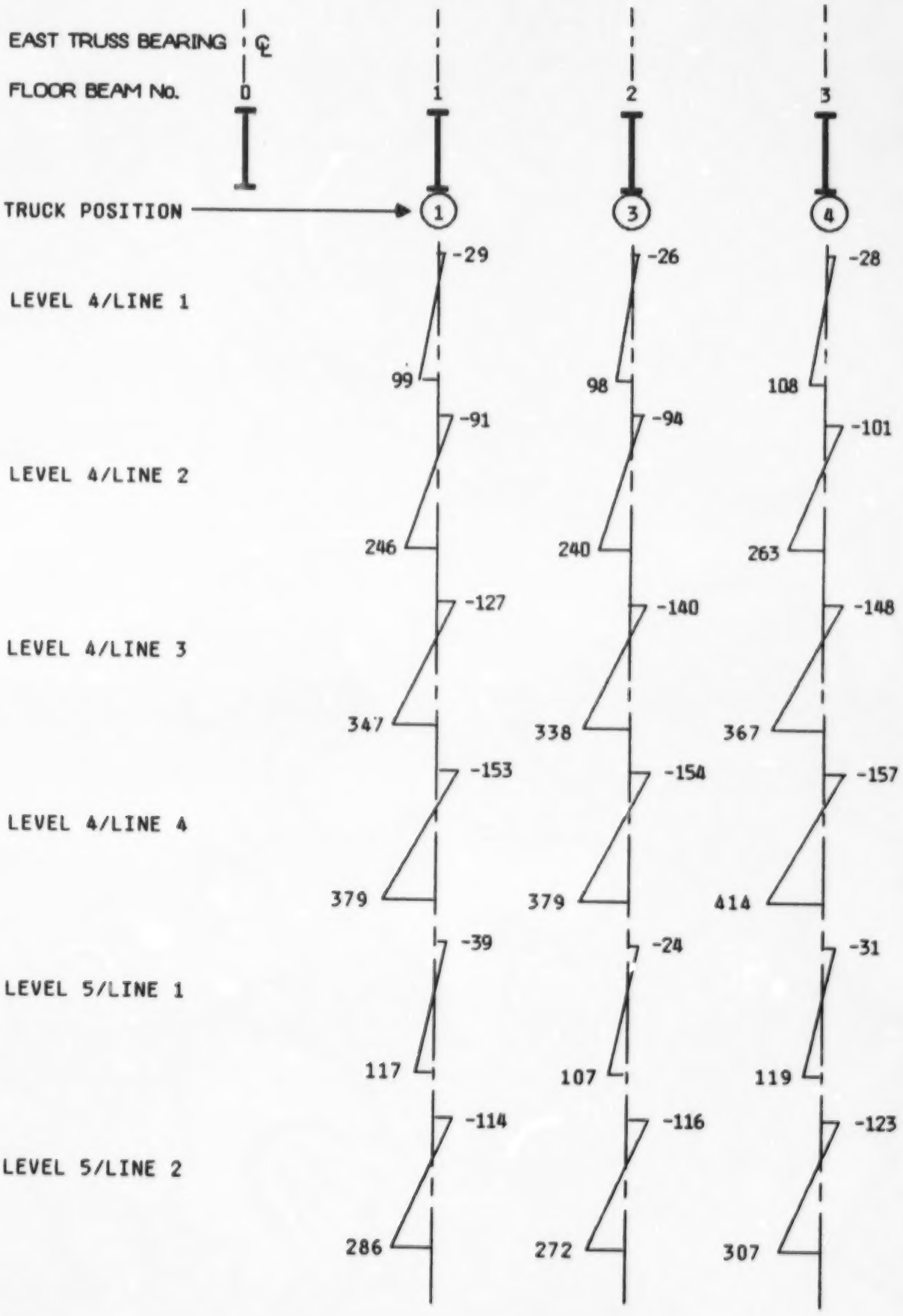


Figure 8/ Typical Strains Measured On the Floor Beams

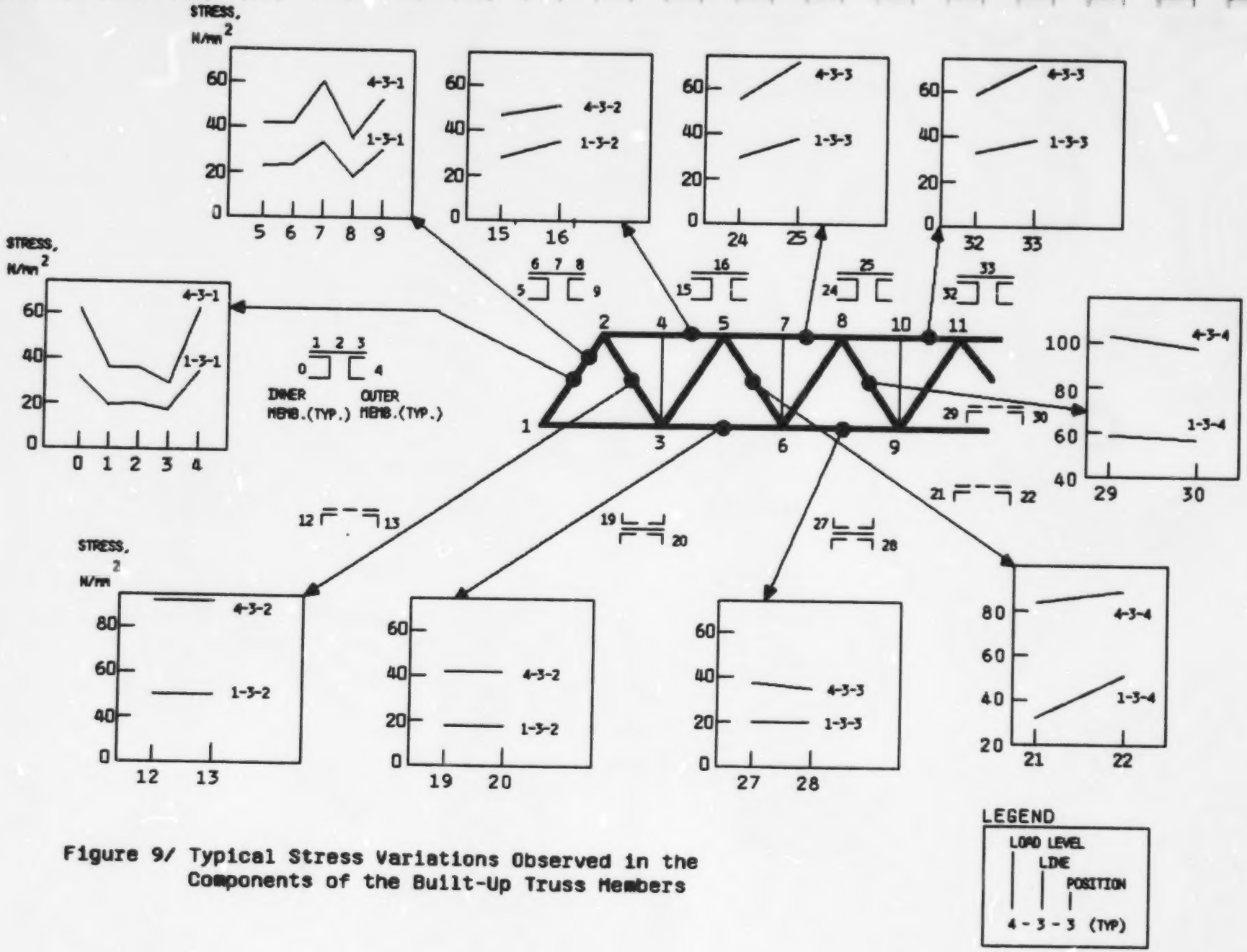


Figure 9/ Typical Stress Variations Observed in the Components of the Built-Up Truss Members

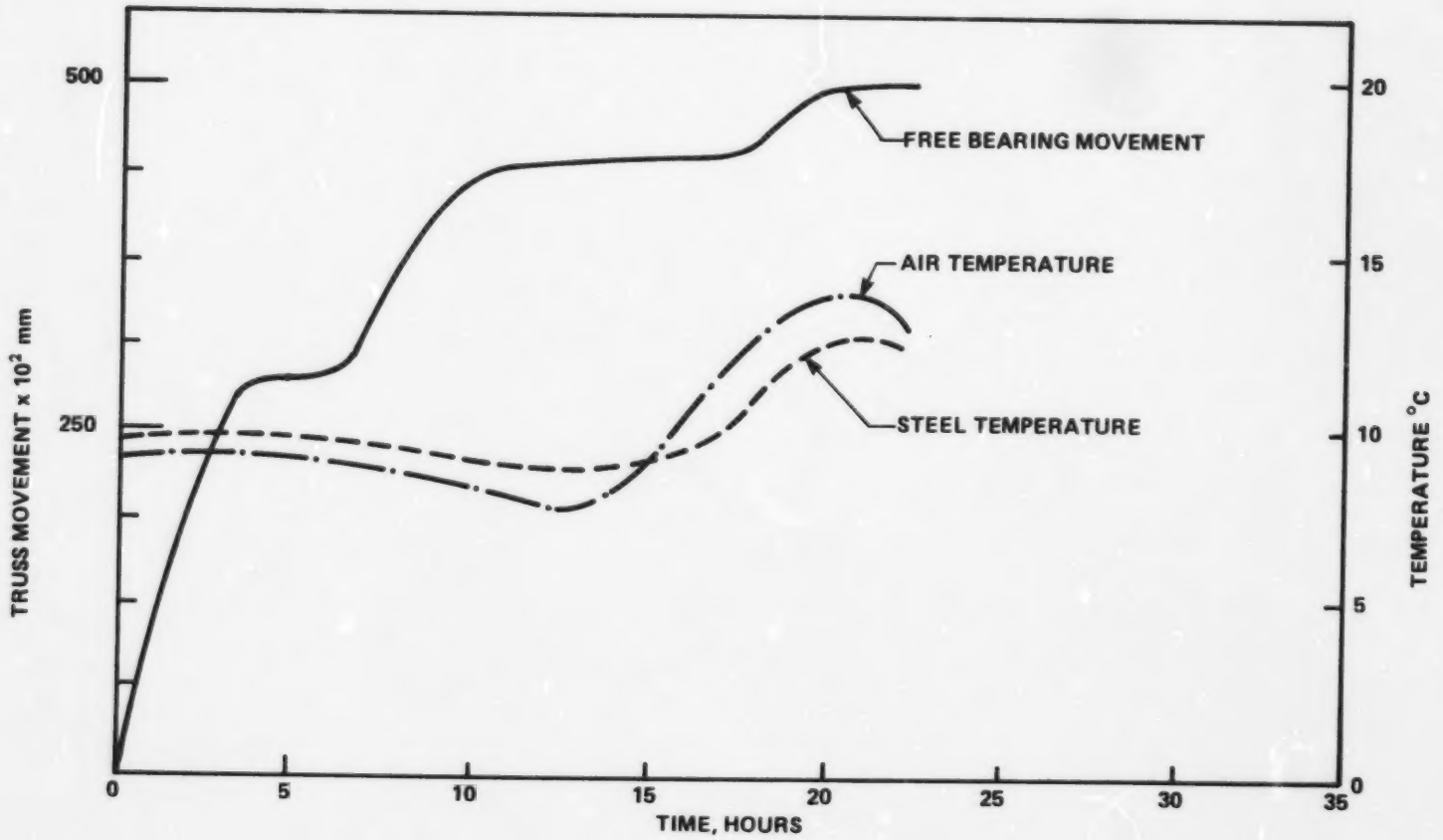


Figure 10/ Temperature Movement Related to Truss Bearing (Free) Movements